International Journal of Engineering Research & Technology (IJERT)
ISSN: 2278-0181
Vol. 3 Issue 1, January - 2014

Effects of Stabilized Calcrete as Sub-Base Material

[1]Aneke Ikechukwu Frank, [1]Agbenyeku Emem-Obong Emmanuel

[1]*Department of Civil Engineering Science, University of Johannesburg, P.O.Box 524, 2006, South Africa*

Abstract

Road construction has always been considered to be an expensive project around the world and governments have researched on ways of minimizing costs of pavement construction. Bulk of road construction cost lies heavily on the cost of materials and the transportation of imported materials required to meet standards for specific pavement construction. This paper looks at how local materials are improved for use in construction of low traffic volume roads. The materials are improved by means of chemical stabilization in conformance to international standard specification. Calcrete a locally available aggregate used in this study were collected from Kwazulu Natal region, South Africa. Results from laboratory tests on stabilized calcrete showed the product to be unsuitable as sub-grade material in low traffic volume road due to high plasticity and low strength factor. Strength of sub-grades was measured by the resistance to deforming loads when vehicular axle load was exerted on the specimens translated in the laboratory by measuring the California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) of the product.

1. Introduction

The use of non-standardized materials in road base construction is utilized in most parts of the world. These materials have been seen to be unsuitable as sub-grades if not stabilized. Previous works show the use of non-standardized materials stabilized by lime (L) and fly ash (FA) to meet standards for sub-grades in pavement construction. This study was therefore motivated by the reduced cost effect of stabilized non-standardized materials in pavement construction. In large areas of most tropical countries the lateritic fine grained soils are the most frequent local materials found along road routes. However, the use of these soils in pavements has been difficult because they do not conform to the conventional specifications for base courses. A paper by [1] on the study of the successful use of these soils as pavement base courses following specially developed geotechnical test methods and construction procedures thus, reducing the cost of pavement construction mainly for low to medium traffic levels were studied.

Currently in South Africa, calcretes are used in construction of road because of its engineering properties. However, the engineering properties of calcretes depend on a generalized manner, basically on two factors;

- The nature of the host or parent material i.e. whether it was clay or sand, and
- The stage of development of the material i.e. the extent to which it has been cemented or replaced or both by carbonate.

In view of the above, the properties of calcareous soils are dominated by those of the host soil whereas, at the other extreme, hardpan calcretes behave like limestone. Therefore, making some of the properties of calcrete which are noteworthy particularly with regard to the specification for their use in road construction and their performance in service [2].

2. Material and Method

The properties of the material were determined by standard tests for selection purposes. In certain cases, additional tests were required which were important in the classification of specific material types. Standard procedures exist in most countries (e.g. BSI, ASTM or SABS) although slight variations may occur.

The tests that were applied to the pavement materials in this investigation are;

- Particle size analysis………{T M H 19}
- Consistency tests ………….{T M H 19}
- Mineralogical analysis…….{S A B S}
- Strength tests ……………...{A S T M}
- Durability tests ……………{A S T M}

- **Particle size analysis**

The sieve analysis was a basic test consisting sieving measured quantities of soil through series of successively smaller sieves. The weight retained on each sieve was expressed as a percentage of the total sample.

- **Consistency tests**

As moisture was removed from the fine-grained soil by passing through series of states i.e. liquid, plastic, semi-solid and solid, the moisture contents of the soil at the points where it passes from one stage to the next were its consistency limits. These limits were further defined by the following;

Liquid Limit (LL): This was the minimum moisture content (MC) at which the soil flowed under its own weight. The LL of the aggregate was the MC expressed as percentage of the mass of the oven-dried aggregate at the boundary between the liquid and plastic states. The MC at this boundary was arbitrarily defined as the LL and is the MC at a consistency determined by means of the standard LL apparatus.

Plastic Limit (PL): The PL of the soil was the MC expressed as percentage of the mass of the oven-dried soil at the boundary between the plastic and semi-solid states.

Shrinkage Limit (SL): The maximum MC was the point at which further loss of moisture does not cause decrease in the volume of the soil.

- **Mineralogical analysis**

This included tests that accurately defined the mineral constituents and contents of the different aggregate samples used in this study. This was achieved by carrying out chemical composition tests, clay mineralogy analyses and petro-graphic analyses.

- **Strength tests**

The strength tests done in this paper included the Mod-AASHTO compaction, CBR and UCS tests. They were carried out to provide appropriate guidelines for field works.

- **Durability tests**

Soil stabilizing agents such as cement has long been used to improve the handling and engineering characteristic of soil which ensures that the mixture meets certain durability requirements for civil engineering purposes. The use of soil-cement as a paving material in the construction of low cost roads dates back to 1920 when the State Highway Department, USA built short sections of roads with soil-cement. To date, thousands of kilometers of road bases have been laid. The performance of soil-cement bases and sub-bases has been proved to be more than satisfactory by various research workers and agencies as in [3].

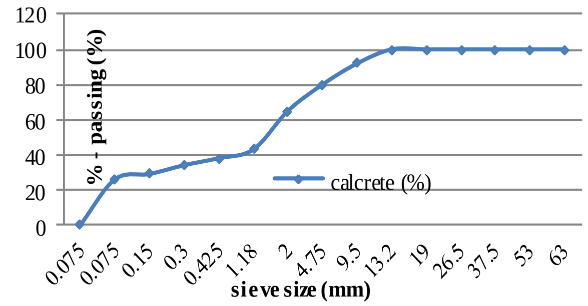

Figure 1: Particle size curve of calcrete

The study herein looked at the pressing need to define a lime-stabilized mixture designed and tested protocol which accounted for durability. The investigation assessed the proposed designed mixture protocol to be established for the desired engineering properties on the selected soils.

Table 1: Geotechnical properties of calcrete

Specimen	Calcrete
Plastic Limit (PL)	40%
Liquid Limit (LL)	73%
Plasticity Index (PI)	33%
Major minerals	Kaolin,quartz,gypsum,calcite

Table 2: Composition of stabilized aggregates

	SiO_2	Al_2O_3	Fe2O3	MnO	MgO	CaO	Na_2O
EM3	65.02	11.07	6.00	0.0890	6.23	8.19	0.51
EM4	2.33	0.19	0.27	1.128	1.65	93.87	0.00
EM5	12.57	1.15	0.71	0.0744	3.37	81.29	0.15
	K_2O	TiO_2	P_2O_5	Cr_2O_3	NiO	Total	L.O.I
EM3	1.89	0.9181	0.03	0.0598	0.0088	100.02	25.53
EM4	0.06	0.0213	0.02	0.0015	0.0000	99.55	31.14
EM5	0.21	0.1187	0.02	0.0107	0.000	99.66	42.23

Where, EM3: calcrete, EM4: hydrated lime, EM5: kaolinite (residual soil fines)

This was representative of typical candidates for lime stabilization. The proposed mixture analysis protocol was designed to produce mixtures that possessing significant structural properties desirable in pavement layer. Figure 1 shows the particle size distribution for calcrete aggregate and Tables 1 and 2 displays the major minerals and geotechnical properties of calcrete and the chemical composition of stabilized aggregate respectively.

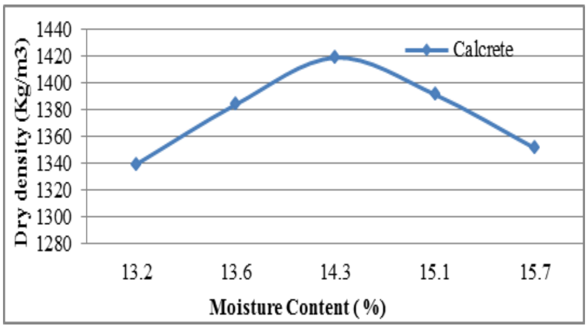

Figure 2: Compaction curve for calcrete

Figure 2 shows the compaction curve for the calcrete aggregate whereas Tables 3 and 4 shows the shrinkage limits and CBR values for the unstabilized calcrete respectively.

Table 3: Shrinkage limits for calcrete

	CAC01	CAC02	CAC03
Day 1	17.95	22.65	5.29
Day 2	17.98	22.71	5.31
Day 3	18.32	23.05	5.54
Day 4	18.39	23.13	5.89
Swell, S (%)	0.346	0.378	0.427

Table 4: CBR for unstabilized calcrete

Sample No.	Load (KN)	CBR (%) at 2.54mm	CBR (%) at 5.08mm	CBR (%) at 7.62mm
CAC01	1.28	9.57	8.94	7.10
CAC02	1.53	11.49	10.69	9.51
CAC03	1.61	12.07	9.09	7.45

3. Result and Discussion

3.1. Penetration effects on calcrete samples

Figures 3 to 5 show graphical representations of the CBR results for the calcrete aggregates. It can be seen that as the load increases the depth of penetration also increases across the different samples. The CAC03 sample was observed to have the highest load against the lowest penetration.

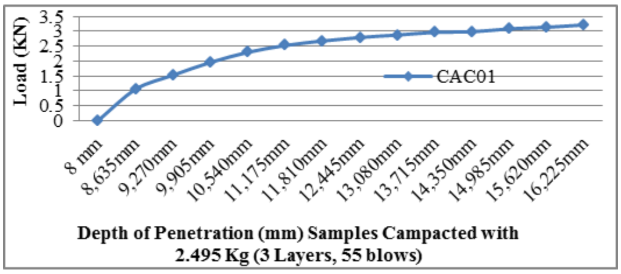

Figure 3: CBR curve for calcrete (CAC01)

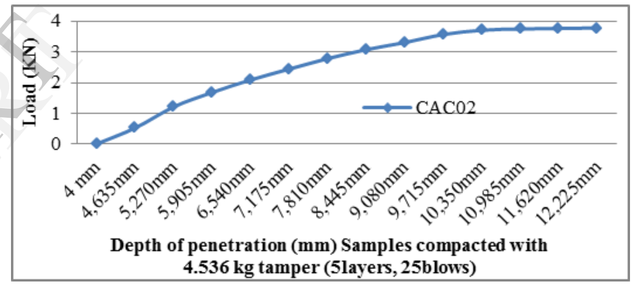

Figure 4: CBR curve for calcrete (CAC02)

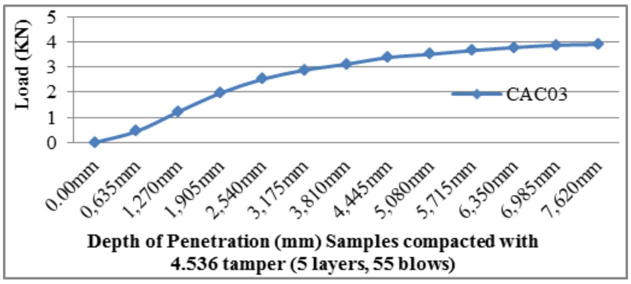

Figure 5: CBR curve for calcrete (CAC03)

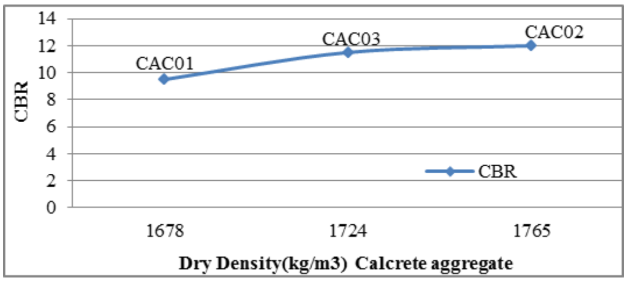

Figure 6: CBR-density relationship for calcrete

The CBR-density relationship for the different calcrete aggregate samples is shown in Figure 6. It was revealed that the highest CBR value of about 12% was achieved for the CAC02 calcrete sample. Values of CBR for the respective treated samples and their graphical expressions are given in Table 5 and Figure 7 respectively.

Table 5: CBR for treated calcrete aggregate

Penetration (mm)	Load (KN)					
	CAC 04	CAC 05	CAC 06	CAC 07	CAC 08	CAC 09
0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.64	0.62	0.80	0.94	1.05	0.90	1.05
1.27	1.34	1.77	1.95	2.16	1.85	2.12
1.91	2.34	2.89	3.10	3.51	3.01	3.20
2.54	3.66	4.19	4.54	4.91	4.21	4.42
3.18	5.09	5.71	6.15	6.45	5.53	5.77
3.81	6.11	6.90	7.26	7.61	6.52	6.71
4.45	6.96	7.68	8.00	8.18	7.01	7.16
5.08	7.47	8.05	8.46	8.40	7.25	7.41
5.72	7.75	8.34	8.78	8.67	7.43	7.58
6.35	7.92	8.55	8.95	8.81	7.55	7.71
6.99	8.04	8.65	9.05	8.90	7.63	7.79
7.62	8.09	8.70	9.10	8.92	7.65	7.81
CBR (%)	27.43	31.40	34.02	36.80	31.55	33.12

It can be seen that the curves show a clear trend of increasing load as the penetration depth increases. This trend was recognized across the entire treated samples.

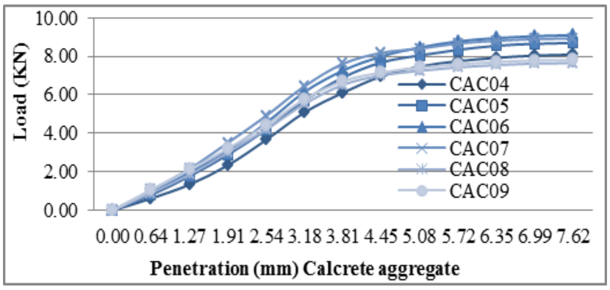

Figure 7: CBR curves for treated samples

3.2. Stabilizer effects on 7days UCS

In Figure 8, the 6% silica additive was found to give the highest strength effect over the 12% silica content. A peculiar change was however observed with a reduction in strength just after the 12% kaolinite content for both the 6% and 12% silica additives.

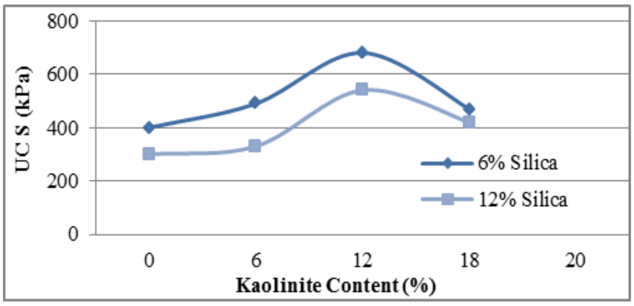

Figure 8: Effect of kaolinite content on UCS

There was an increased development of strength in samples with 6% silica additive up to 18% cement content. However, the strength of 12% silica additive was found to increase to the 12% cement content after which a rapid decrease in strength was observed.

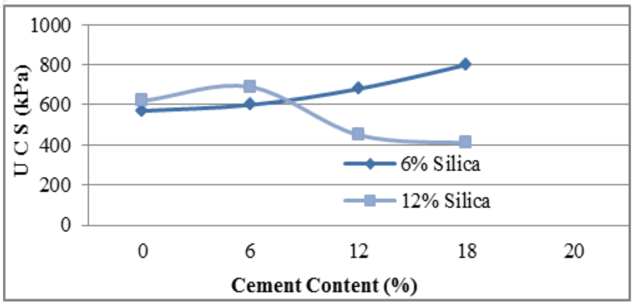

Figure 9: Effect of cement content on UCS

The reduction in strength can be attributed to the excess binder in the sample which was idle in the hydration-cementing reaction. Figure 9 therefore shows the graphical expression of the effect of cement content on the strength of silica treated samples.

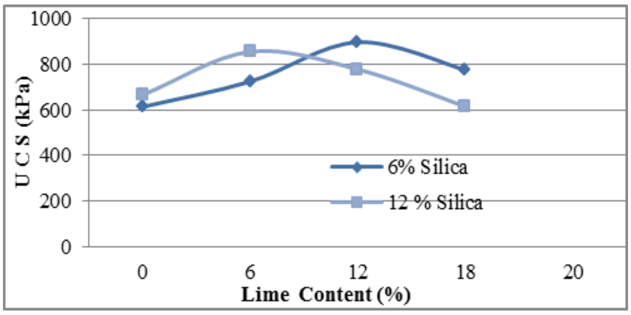

Figure 10: Effect of lime content on UCS

A similar strength gain was recorded for the 6% silica-lime treated samples. This was the case until the 18% lime content where a reduction in strength was observed. As for the 12% silica-lime treated samples, there was strength increase till the 6% lime content after which a drop in strength was noticed. The reduction in strength could be as a result of loss in material contents of the samples and the excess binding materials not taking part in the cementing process as shown in Figure 10.

3.2. Stabilizer effects on 14days UCS

The results of samples of 6% and 12% silica additives as shown in Figure 11 were found to exhibit similar trends. The kaolinitic contents at 6% and 18% lead to a drop in the treated samples but an appreciable increased strength was observed at the 12% content. However, the 6% silica additive was found to have the highest UCS across the entire samples.

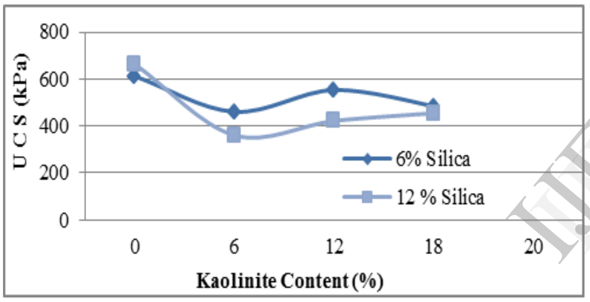

Figure 11: Effect of kaolinite content on UCS

The cement treated samples for both the 6% and 12% silica additives showed varying results with increasing cement content. The samples of 6% silica additive were found to be higher than the 12% silica additive.

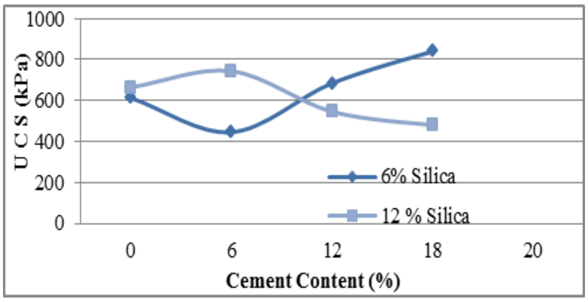

Figure 12: Effect of cement content on UCS

The reduction in UCS for the 12% silica additive with increasing cement content can be due unused idle cementing materials not taking part in the hydration processes involved as seen in Figure 12.

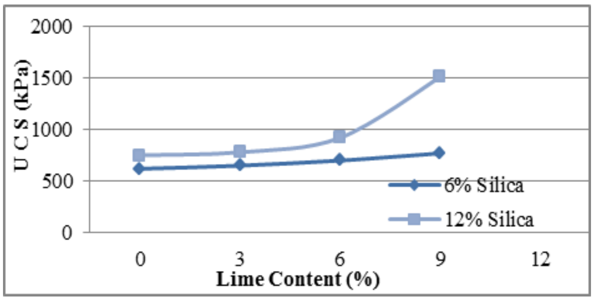

Figure 13: Effect of lime content on UCS

The UCS results of treated samples in Figure 13 shows that the 12% silica additive behaved better than the 6% silica additive. Nevertheless, a continued increased strength gain was observed for both silica percentages up to the 9% lime content in the treated samples.

3.3. Stabilizer effects on 28days UCS

The 28day UCS results in Figure 14 shows a gradual strength increase with increasing cement content for both the 6% and 12% silica additives. The 12% silica treated samples were found to have the highest UCS as compared to the 6% silica additive. The increased strength in the 12% silica can be associated with the increased hydration periods of up to 28days.

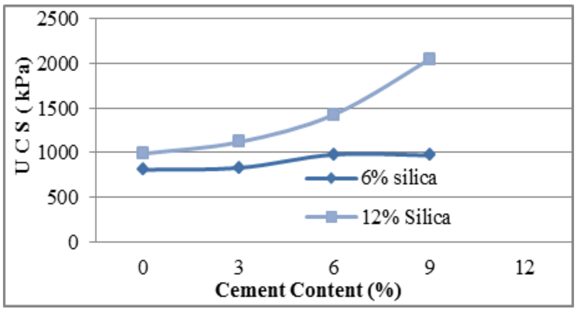

Figure 14: Effect of cement content on UCS

The kaolinite treated samples having both the 6% and 12% silica additives were found to have improved strength gain.

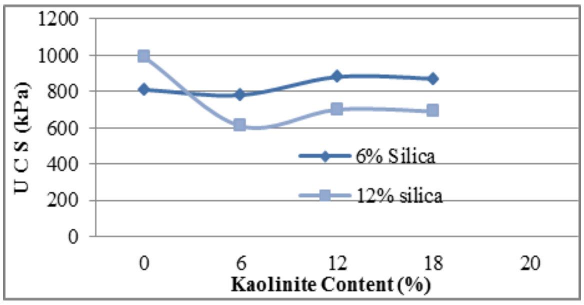

Figure 15 Effect of kaolinite content on UCS

However, strength loss at the 6% kaolinite content for the 12% silica additive was recorded. The kaolinite content effects on the UCS of the treated samples are shown in Figure 15 with samples of the 6% silica additive showing the best strength behaviors.

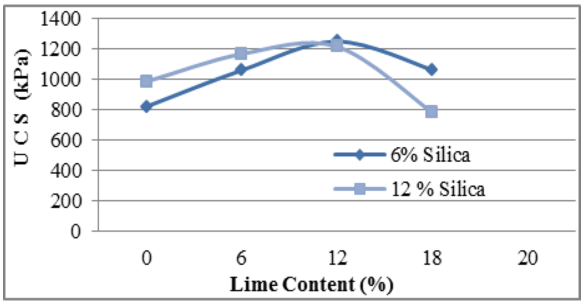

Figure 16: Effect of lime content on UCS

A simultaneous strength increase in the lime treated samples for both the 6% and 12% silica was displayed as shown in Figure 16. Reduction in their respective strengths was however noticed after the 12% lime content was added.

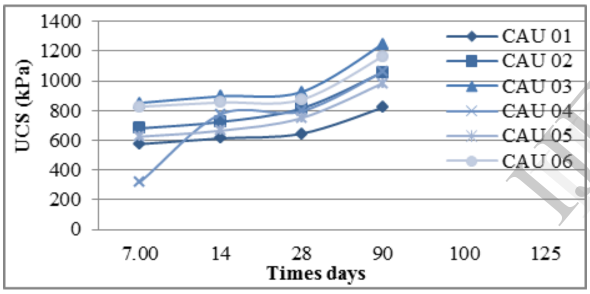

Figure 17: Effect of curing on UCS of treated calcrete

The hydration effect on the strength of the tested materials for the various treated samples as seen in Figure 17 showed a general increase in the UCS over the respective curing ages. The increased general rise in strength can be attributed to the extended hydration effects on the samples.

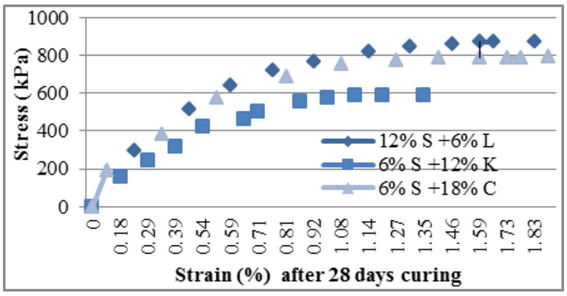

Figure 18: 28day curing stress-strain relationships

The stress-strain relationship for the 28days curing is shown in Figure 18. It can be seen that as the stresses induced in the samples increases their strain values also increases. This trend was the case across the different tested samples.

3.4. Durability of cured calcrete samples

The durability of stabilized soil is primarily evaluated from the degree to which the engineering properties are retained. Although dominating exposure conditions may vary geographically but such exposures are common. One of the most commonly used durability test soil-cement-lime mixes in a non-frost area is wetting and drying test. The durability test results for the 7days curing age are shown in Table 6.

Table 6: 7days durability results for calcrete

SAMPLE	CA D01	CA D02	CA D03	CA D04	CA D05
Maximum Dry Density(Kg/m^3)	1324	1289	1204	1268	1235
Optimum Moisture Content	20.0	20.6	20.9	21.4	18.5
Compactive Effort (%)	99.7	99.8	99.8	99.7	99.9
Mass after compaction(g)	3668	3575	3342	3516	3430
Stabilizer Loss after 1 cycle	5.5	6.0	4.7	2.2	3.3
Stabilizer Loss after 2 cycles	10.5	9.3	11.5	3.9	6.5
Stabilizer Loss after 3 cycles	17.4	14.4	18.9	5.4	8.3
Stabilizer Loss after 4 cycles	23.7	21.2	23.2	8.0	9.4
Stabilizer Loss after 5 cycles	25.2	24.9	29.4	15.5	11.6
Stabilizer Loss after 6 cycles	25.2	29.5	36.9	23.1	16.4
Stabilizer Loss after 7 cycles	27.8	31.1	41.7	27.3	27.3
Stabilizer Loss after 8 cycles	31.4	33.0	45.4	32.1	35.4
Stabilizer Loss after 9 cycles	31.8	34.8	47.6	37.1	39.1

Stabilizer Loss after 10 cycles	34.6	36.9	49.8	38.4	43
Stabilizer Loss after 11 cycles	35.6	38.7	51.6	40.6	44.8
Stabilizer Loss after 12 cycles	36.3	40.6	53.0	42.3	45.6
Mass after 12 cycles,M_4(g)	2338	2254	1570	2668	2644
Final Mass, M_{12}(g)	2932	2814	1966	3334	3274
Final Mc (%)	25.4	24.8	25.2	25.0	23.8
Final Diameter (mm)	134	126	124	135	139
Final Height (mm)	123	115	112	122	121
Final Volume,$V_{12}(m^3)$	0.00173	0.001434	0.00135	0.001746	0.001836
Volume change (%)	37.27	48.15	51.09	36.85	33.60
Change in mass, Mc (%)	0.363	0.406	0.530	0.423	0.456
Loss of mass, M_A(%)	36.3	40.6	53.0	42.3	45.6
Change in Mc (%)	27.0	20.6	20.7	16.6	28.8

Where; Calcrete Aggregate
Sample Number Mix_Proportion

CAD01	6%S+6%L
CAD02	12%S+6%L
CAD03	18%L+0%S
CAD04	6%L+6%K+6%C
CAD05	6%S+6%K+6%C

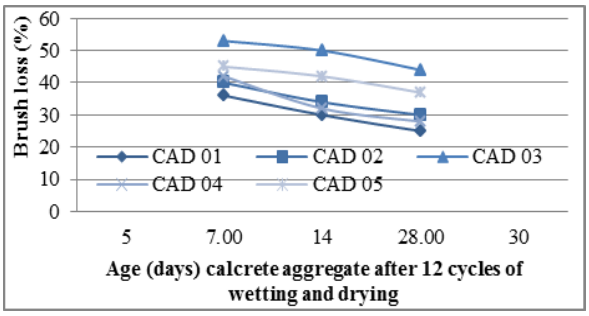

Figure 19: Effect of curing on brush loss

The wet-dry cycles lead to continuous brush loss with increasing wetting and drying. Most of the durability loss was observed at the 12cycles of wetting and drying. The brush loss can be accounted for by the adsorption of water and simultaneous loss in material constituents of the samples as shown in Figure 19.

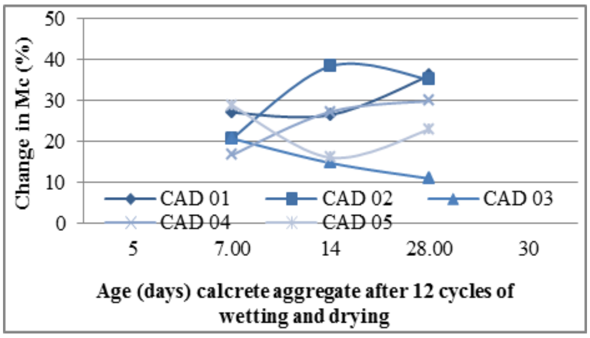

Figure 20: Effect of curing on moisture content

The Figure 20 shows a mixed up behaviour of the samples moisture content with the corresponding 12 cycles wet-dry effect on durability. In some of the samples, increased moisture was observed with subsequent reduction after the 14days period while in other samples, a straight complete reduction or a reduction and subsequent improvement in strength was recorded.

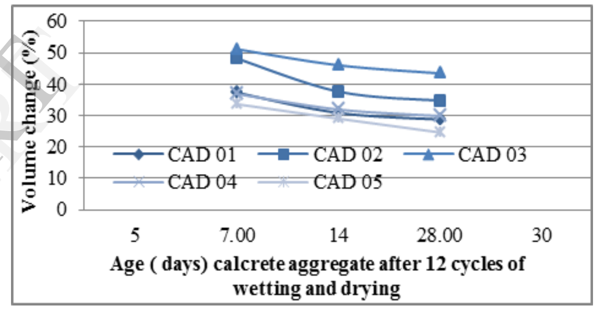

Figure 21: Effect of curing on volume change

The increased reduction in volume change recorded from the wet-dry cycles for the treated samples was a general trend across all samples at the 12 cycles. The volume change can be accounted for by the loss of constituent materials from the repeated wetting and drying of the respective samples. This material loss eventually led to reduced specimen masses, strength, volume and in turn the durability of the product. Figure 21 shows the effect of curing, wetting and drying on the volume change in the treated samples.

Clay soil is a variable and complex material but because of its availability and low cost it is frequently used for constructions purposes. At a particular location, however, a clay soil may not be wholly suitable for the desired purpose. This soil usually possesses medium or high or even very high plasticity and thereby increasing construction difficulties [4]. Problematic soils (swelling or soft) contain clay minerals that exhibit medium to high volume change upon wetting. The large volume change upon wetting causes extensive damage to structures in particularly in light buildings and low volume pavements [5].

The Calcrete aggregate used was well-graded. The sample could be said to be of high plasticity due to the high liquid limit of 73% and a plastic limit of 35%. The plasticity index, Ip was found to be 38. The sample felt plastic when rubbed between the thumb and forefinger and showed considerable shrinkage after drying. The sample can was classified as inorganic clay according to the Casagrande's plasticity chart (CH).

The materials were found to be unacceptable for sub-grade material according to the limits set out in [6].

The X-ray diffraction showed the aggregates to consist of various mixtures of quartz, kaolin, hematite, feldspar, gypsum and calcite. A semi-quantitative estimate of the relative proportions of the major minerals was obtained by carrying out a chemical composition analysis of the weight percentages of the aggregates samples. The percentages showed that the dominant element in the aggregates was silica with relatively lower amounts of alumina. However, the calcrete aggregate had significant layered fractions of MnO and MgO.

From the results we can conclude that the relative high maximum dry density of calcrete sample is 1702 kg/m^3. This could be due to the fact that the former samples had relatively higher amount of fines.

The calcrete sample's CBR value is the index of its resistance to shearing under a standard load compared to the shearing resistance of a standard material subjected to the same load. The test carried out was for the determination of the initial design CBR value and together with the grain size distribution values and Atterberg limits, the final design CBR was then determined.

A general trend is observed in the results obtained from the aggregates with the silica and the lime mixtures. However, the initial addition of silica increased the CBR value. This is attributed to the high clay content in the soil which increases the pozzolanic reactions between the hydrated lime and the alumina content present in the clayey aggregate.

The calcrete aggregate samples portrayed evidence of absorption of calcium cation (provided by the lime) at the surface of the clay particles in place of the commonly present cation in the natural clay which resulted in a cation exchange or molecular crowding.

The saturation of calcium or calcium hydroxide molecule at the clay surface dramatically reduces the energy with which the clay surface attracts and holds water. As a result, the physical changes in the silica-lime, silica-kaolinite and silica-cement soil systems are:

- Flocculation and agglomeration of particles into a larger effective particle size
- Reduction in plasticity and swell potential
- A drying effect as the water-holding potential of the soil is reduced
- Improved workability and compatibility
- Improved aggregate crushing strength as a result of the cation or molecular crowding

The aggregates also portrayed a reaction between the calcium in the lime and the silica and alumina released from the aggregates (particularly Calcrete) in the stabilization process. When the required amount of lime is added to a soil, pH increases, which allow the soil alumina and soil silica (pozzolans) to become soluble and hydrates (CSH), while calcium-aluminates-hydrates (CAH).

The high pH at the soil surface allows the lime to actually solubilize silicates and aluminates from the surface. This alters the mineralogy of the aggregate causing a reduction in plasticity and an increase in strength which continues for an extended period of time. As a result of this reaction, the following physical benefits are derived:

- Further reduction in plasticity and swell potential as the time-dependent pozzolanic reaction proceeds
- Further increase in effective particle size and improved workability as the pozzolanic reaction proceeds
- Substantial increase in aggregate crushing strength as the pozzolanic reaction proceeds.

Carbonation reactions were also evident in the aggregate samples were calcium reverted to calcium carbonate as free lime reacted with carbon dioxide from the atmosphere. This reaction is for free calcium needed in the stabilization process.

However, works by [7] has demonstrated the benefits of the carbonation reaction over the long-term in the stabilization of calcareous aggregates.

In calcareous material, lime has been shown by [8] to enhance the growth of carbonate cement which bonds carbonate particles together resulting in substantial shear strength and a substantial stiffness increase.

Form the durability result, kaolinite present in the stabilized aggregate samples is damaged by high temperatures. The behaviour of kaolinite is characterized as inert and generally develops satisfactory strength and durability with economical amounts of cement after short curing periods (7-28days).

In relation to the acceleration of the climatic differences in the wetting and drying durability test, the development of the strength and durability of the stabilized aggregates depends upon the effectiveness of the stabilizers on the mineralogical composition of the soils for the increased strength and durability of stabilized aggregates.

Strength increases with the increase of stabilizer content and the increase in stabilizer loss in the durability test is connected to the weak bonds of stabilizer-soil-sand systems due to high content of lime and kaolinite. The above observations indicate that for safety reasons in construction works the durability should be considered as a serious factor together with strength.

Conclusion

From the results we can conclude that the stabilized calcrete samples reflect relatively high maximum dry densities. However major conclusions arrived at in this study include;

i. The materials sampled had different engineering properties suitable for low moisture compaction.
ii. Linear shrinkage was a more consistent parameter for determining the moisture susceptibility of calcretes.
iii. Calcrete aggregate are strong and appreciably durable enough for use in low volume roads.
iv. Addition of lime, silica and kaolinite respectively to calcrete material increased compacted strength, decreased placidity and improved grading.
v. The relevant parameter needed in determining the suitability of calcrete as pavement material is CBR.

Generally, it shows that the calcrete materials can be considered for low moisture compaction which reduces the road construction cost.

Acknowledgement

The Authors appreciate the University of Johannesburg where this investigation was successfully carried out.

References

[1] Nogami J.S and Valibor D.F 1995. Low Cost Pavement with Lateritic Soils (in Portuguese). Vilibor, Sao Paulo 213pp.
[2] Netterberg 1969. General Characteristic and Properties of Calcrete.
[3] AASHTO and Larsen 1967. The Performance of Soil-Cement Bases and Sub-Bases.
[4] Bell and Tyrer 1989. Effect of Incorporating Stabilizers in the Soil Stabilization Process on the Engineering Properties of Soil.
[5] Fredlund D.G. and Rahardjo H. 1993. Soil Mechanics for Unsaturated Soils. John Wiley, New York.
[6] National Institute for Transport and Road Research, Structural Design of Interurban and Rural Road Pavements, 1985. TRH 4, Pretoria South Africa.
[7] Bowels J.E. 1983. Engineering Properties of Soil and their Behavior.
[8] Graves and Smith L.L. 1996. Carbonation Cementation of Silica, Carbonation of Base Course Material Transportation Record no. 1190.